IN THE UNITED STATES PATENT AND TRADEMARK OFFICE

Application No.	:	09/639,599
Applicant	:	Alex S. Toback
Filing Date	:	August 16, 2000
Title	:	Connection System for Steel Construction
TC/A.U.	:	3726
Examiner	:	Essama Omgba
Docket No.	:	TOB/102/US
Customer No.	:	002543

Mail Stop Amendment
Commissioner for Patents
United States Patent and
Trademark Office
P.O. Box 1450
Alexandria, Virginia 22313-1450

Sir:

DECLARATION UNDER 37 CFR §1.132

I, Alex S. Toback, hereby declare and state that:

1. I am Technology Consultant to Henkel Corporation of Rocky Hill, Connecticut and I am the named inventor of the above identified U.S. Patent Application.

2. I am presenting this Declaration to establish facts which support the patentability of the pending claims of the above identified application.

3. I have had extensive international, general management, industrial distribution, technical, manufacturing and marketing experience in the specialty chemical and fastener industries. Since 1995 I have been very actively involved in designing fasteners and fastening systems for the light gauge steel industry and in particular fastening systems which are employed in light gauge steel construction for light commercial and residential applications.

4. I have extensive knowledge and background in both the mechanical and the chemical fastening technologies and have used that background and knowledge to address the issue of providing fastening systems for the light gauge steel industry. A true copy of a resume of my educational, managerial and technical career in the specialty chemical and fastener industries is attached to this declaration as Exhibit A.

5. It is very apparent in reviewing the Office Action dated February 8, 2005, and in particular the portion commencing with page 2, paragraph 3 that the rejection of the claims is based on an inaccurate, very incomplete appreciation of the level and knowledge of one of ordinary skill in the light gauge steel construction industry at the time of my invention.

6. As stated in the Patent Specification and as referenced by the Examiner, at the time of the invention of the pending claims 1-24, it was known in the light gauge steel construction technologies to provide a light gauge steel construction and assembly wherein numerous self drilling screws or other fasteners were used to provide a connection between a panel and a support structure (the Examiner refers to the latter as "AAPA"). It was in this context, as President of Metaltite

Corporation, a specialty mechanical fastener corporation, that I turned my attention to the fastening systems for the light gauge steel industry and my original focus in this regard was purely in terms of mechanical fasteners.

7. At the time of my invention of the pending claims of the patent application, a person of ordinary skill in the light steel construction industry was wholly focused on mechanical fasteners and how to provide an improved mechanical fastening system. Until I started to consider addressing the light gauge steel construction fastening systems, there was no thought or suggestion whatsoever in the light gauge steel construction technology of using chemical or adhesive systems in light gauge steel construction. One important factor in assessing the level of ordinary skill and knowledge is that the light gauge steel construction industry typically must comply with building and fire codes in connection with the integrity of the fastening systems, and these codes were and typically now are all based on mechanical fastening systems.

8. Therefore, the statement by the Examiner "Therefore it would have been obvious to one of ordinary skill in the art at the time the invention was made, to have used an adhesive material in the connection of AAPA, in light of the teachings of Orowan, in order to relieve the load on fasteners to a relatively small extend and give protection against threading of the parts joined" fails on two grounds. First, there is no objective evidence that it would have been obvious in any way to even attempt to use an adhesive system for light gauge steel construction. Second, the problem addressed was <u>not</u> "to relieve the load on fasteners to a relatively small extent and give protection against fretting between the parts joined." The defined problem which was addressed by the invention of claims 1-

24 of the present application was to provide a fastening system of high integrity which could be implemented in a highly efficient and cost effective manner through improvement of mechanical fasteners and reduction of labor costs since labor is such a significant factor in the cost of the light gauge steel construction. This solution was realized by the use of both mechanical fasteners and the adhesive systems as set forth in the claims 1-24 of the pending application.

9. Contrary to the assertion of the Examiner, there is no evidence that one of ordinary skill in the light steel construction industry would have turned to the Orowan reference. The Orowan reference when carefully examined by one of ordinary skill in the light gauge steel construction industry would be deficient for the following reasons: First of all Orowan focuses on, as acknowledged by the Examiner, a protection against fretting between parts joined. This was not a problem presented in the light gauge steel construction technology at the time of the invention. The present invention is concerned with high strength assemblies which under stress in almost all cases result in failure of steel components rather than the connections. Second, the solution by Orowan proposes a relatively expensive and complex strip of adhesive tape which simply would not be suitable for the application as recited. Third, the Orowan reference does not, considered as a whole, result in a fastening system where the assembly is substantially enhanced in load-bearing capacity compared to that of the mechanical fasteners alone as provided in the assemblies of claims 1-24. Fourth, the passage at col. 1, lines 24-29 does not support the conclusion that "the connection of Orowan is significantly enhanced in load bearing capacity".

10. Contrary to the apparent belief of the Examiner, the invention by the undersigned applicant is not a mere paper concept, but is in fact an invention which has been tested by experts in the light gauge steel construction industry and has great potential as an industry standard. A true copy of a report submitted to the applicant which shows test results on some applications of the applicant's claimed system is attached hereto as Exhibit B.

11. As a result of the invention embraced by the pending claims of the application, I have been contacted by numerous prominent companies and organizations in the light gauge steel industry to discuss the invention and further development of the invention for specific light gauge steel applications. I have been requested to make presentations at industry technical meetings and trade shows. On October 20, 2004 the Light Gauge Steel Engineers Association awarded me a certificate in recognition of outstanding contributions to the advancement of adhesives for the Steel Framing Industry.

I hereby declare that all facts and statements made above are true and accurate to the best of my knowledge and belief and that any opinions are true to the best of my knowledge and belief, and any willful and false statements will jeopardize the validity of any patent issued in the above identified application and may subject me to prosecution under federal law.

Respectfully submitted,

By: ______________________

Alex S. Toback

Date: May 9, 2005

GDY/tlc

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ALEX S. TOBACK

MECHANICAL/CHEMICAL FASTENING SYSTEMS

Extensive international, general management, industrial distribution, technical, manufacturing and marketing experience in the specialty chemical and fastener industries.

Present President, Toback and Associates

Consultants in the specialty fastener field, combining mechanical and chemical systems that result in high quality, cost effective connections.

1993-1999 President, Metaltite Corporation

A "start up" company that develops and markets specialty fasteners, priced to value, that solve assembly problems. Inventor on one U.S. patent and two provisional patent applications.

1991-1995 President, COO The Eagle Group, Inc.

A 77 year old construction firm specializing in roofing, sheet metal and waterproofing in the commercial market.

1985-1991 Vice President, Industrial Distribution The Loctite Corporation

Developed and implemented policy, strategy and tactics for Loctite's extensive business through its industrial distributors in the U.S., Canada and Mexico.
Managed and coordinated all aspects of major distributor accounts.
Doubled sales in the industrial distributor channel.
Directed Technical Sales and Industrial Distributor Training.

1984-1985 Vice President The Loctite Corporation

Developed and implemented the business plan that significantly grew sales in major product line that had exhibited no growth during the previous three years.

1981-1984 Vice President and General Manager Loctite Brasil Limitada, Sao Paulo, Brasil

Increased sales revenue by 15, 25 and 40% during the three year period despite severe economic conditions and aggressive competition.
Achieved operating profit goals.
Restructured a major part of the organization.
Built and moved into an 86,000 square foot manufacturing facility.

1978-1981 Vice President, Technical Loctite International

Provided technical expertise on all major international projects in the Far East, Europe and South America.
Provided extensive technical support in the formation of the joint venture with the Peoples Republic of China.
Managed construction of a specialty chemical plant in Mexico, on time and on budget.
Reduced export staff by 25% while maintaining same level of service to our customers.

1963-1978 Loctite Corporation

Held various middle management positions in R&D, Manufacturing, Technical Service and Training. Developed dozens of new products and inventor of four U.S. Patents.

1962-1963 American Cyanamid Company

As a chemist in R&D, solved major production problem in the manufacture of recently acquired technology.

Education

B.S. Chemistry, Central Connecticut State University 1962(Physics minor)
M.S. Chemistry, Central Connecticut State University 1968

Military Service

U.S. Air Force, 1955-1958

Professional Memberships

Society of Automotive Engineers, Adhesive Seminar Chairman – 9 years: Science Engineering Board Member - 8 years.
American Society for Testing and Materials - 16 years.
American Chemical Society - 28 years
Light Gauge Steel Engineers Association - 4 years.

CENTER FOR LIGHT FRAME STRUCTURAL RESEARCH

Department of Civil Engineering
Santa Clara University

COMBINED ADHESIVE-STEEL PIN APPLICATIONS FOR CFS FRAME SHEAR WALLS

Final Report: CLFSR-05-04

May 31, 2004

By

Ioi Lam, Henry Qi and Christopher Pitt (Undergraduate Research Assistants)
Reynaud L. Serrette (Project Director)

Submitted to

Al Toback
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"Improving Structural Design Through Innovative Applied Research"

Santa Clara, CA 95053-0563

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INTRODUCTION

Over the past several years, a number of innovative proprietary lateral force resisting elements have been developed for implementation in the light frame residential and commercial construction. These elements are typically used where demands are relatively high and wall space is limited. In designs with low to moderate lateral demands, conventional sheathed walls that utilize mechanical fasteners are often sufficient. To explore the potential benefits, both structural and economical, of structural adhesives in combination with mechanical fasteners in light frame shear wall construction, a joint research effort was initiated in 2002 between the Center for Light Frame Structural Research and the Henkel Loctite Corporation.

The use of adhesives to bond sheathing to light framing is not an entirely new concept. In fact, the 2003 IBC, Section 2305.3.9 (wood frame construction) recognizes the additive strength of adhesives but limits the benefits to wind design and structures in Seismic Design Categories (SDC) A, B and C: *"Adhesive attachment of shear wall sheathing is not permitted as a substitute for mechanical fasteners, and shall not be used in shear wall strength calculations alone, or in combination with mechanical fasteners in Seismic Design Category D, E or F."* Section 2305.3.9 imposes no specific requirements on the properties of the adhesive. In the application presented in this report, the role of the adhesive is primary. The adhesive was developed by chemists at the Henkel Loctite Corporation to provide a dependable structural bond between wood structural panels or sheet steel and cold-formed steel framing members with the expectation that the contribution (number and type) from mechanical fasteners may be reduced. Specifically, this report documents the reversed cyclic performance of 27 mil sheet steel and 7/16-in. OSB rated sheathing (24/16 exposure 1) attached to cold-formed steel (CFS) framing with a structural adhesive and pneumatically driven steel pins produced by Aerosmith Inc..

In the following sections, details of the project scope, test procedures and test results are presented, interpreted and discussed.

SCOPE OF WORK

A series of eight single-sided (sheathing on one side only) cold-formed steel frame shear wall tests were conducted on 2 ft. x 8 ft. and 4 ft. x 8 ft. (out-to-out dimensions) walls. The eight tests comprised four

different shear wall configurations that utilized either a single 27 mil (33 ksi) sheet steel or a single 7/16-in. OSB rated sheathing 24/16 exposure 1 wood structural panels.

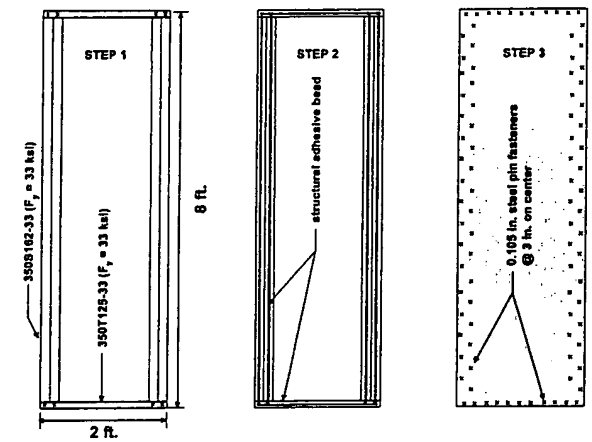
Four of the eight walls were constructed using 27 mil sheet steel. These steel sheathed walls were identical except for their overall dimensions—two walls were 2 ft. x 8 ft. and the other two were 4 ft. x 8 ft. Framing for each wall consisted of 350S162-33 studs at 24 in. on center and 350T125-33 mil top and bottom tracks. The chord studs were back-to-back studs connected with two No. 10 fasteners (transverse to the stud height) at 12 inches on center through the web of the studs. The 27 mil sheet steel was attached to the CFS frame with a bead of structural adhesive on each "contact flange" and Aerosmith 0.105 in. knurled steel pins at 3 in. on center at sheet edges and 12 in. on center in the field. At the chords, the 3 in. on center spacing was achieved with two lines of pins—one line per stud flange—in a staggered configuration. Additional details regarding the configuration of the sheet steel shear walls are given in Table 1 and the sequence of wall construction is illustrated in Figure 1.

The 7/16-in. OSB shear walls were identical (4 ft. x 8 ft.) except for the spacing of mechanical fastener at the panel edges. These walls were framed with 350S162-54 studs at 24 in. on center and 350T125-43 top and bottom tracks. The OSB was attached to the framing with beads of an acrylic structural adhesive on each "contact flange" and Aerosmith 0.105 in. knurled steel pins at either 6 in. or 12 in. on center at the panel edges and at 12 in. on center in the field. The chord studs were back-to-back studs connected with the same structural adhesive used for the sheathing and steel pin at 12 in. on center through the webs. Additional details of the OSB shear walls are given in Table 1 and the sequence of wall construction is illustrated in Figure 1.

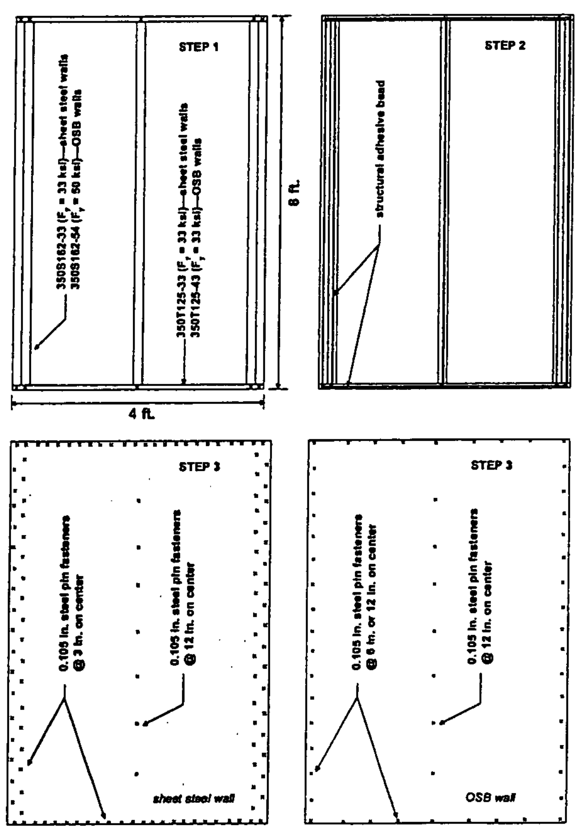
Table 1. Shear wall specimens

Specimen [1,2]	Shear Element	Attachment of Shear Element [3]	Anchorage
2by8-TA 2by8-TB	2 ft. x 8 ft. 27 mil sheet steel (nominal F_y = 33 ksi)	0.105 in. pins at 3 in. on center at the sheet edges and structural adhesive on the contact flange of each framing member	S/HD15 at the chords (back-to-back 350S162-33 studs connected with two No. 10 fasteners at 12 in. on center)
4by8-TA 4by8-TB	4 ft. x 8 ft. 27 mil sheet steel (nominal F_y = 33 ksi)	0.105 in. pins at 3 in. on center at the sheet edges and 12 in. on center in the field; structural adhesive on the attached flange of each framing member	S/HD10 at the chords (back-to-back 350S162-33 studs connected with two No. 10 fasteners at 12 in. on center) and 3/4 in. bolts 12 in. in from each holdown
OSB6-TA OSB6-TB	4 ft. x 8 ft. 7/16-in. (24/16 span rating) OSB rated sheathing	0.105 in. pins at 6 in. on center at the sheet edges and 12 in. on center in the field; structural adhesive on the attached flange of each framing member	S/HD15 at the chords (back-to-back 350S162-54 studs connected with two longitudinal adhesive beads and one steel pins at 12 in. on center) and 3/4 in. bolts 12 in. in from each holdown
OSB12-TA OSB12-TB	Same as above	0.105 in. pins at 12 in. on center at the sheet edges and 12 in. on center in the field; structural adhesive on the attached flange of each framing member	S/HD10 at the chords (back-to-back 350S162-54 studs connected with two longitudinal adhesive beads and one steel pins at 12 in. on center) and 3/4 in. bolts 12 in. in from each holdown

[1] Studs at 24 in. on center
[2] All specimens were 4 ft. x 8 ft. (out-to-out) except 2by8-TA and 2by8-TB which were 2 ft. x 8 ft. (out-to-out)
[3] Nominal adhesive bead width was 0.1875 in.



(a) 2 ft. x 8 ft. walls



(b) 4 ft. x 8 ft. walls

Figure 1. Wall construction sequence

TEST SETUP/PROCEDURE

Each wall was tested in a horizontal position. The bottom track of the wall was attached directly to a reaction beam with holdowns on each end of the wall and 3/4-in. high strength shear bolts 12 in. in from the holdowns (for the 4 ft. x 8 ft. walls only). No shear bolts were used in the 2 ft. x 8 ft. wall tests. The holdown schedule is given in Table 1. With the bottom of the wall anchored in place, the top of the wall was attached to the load distribution member, through the wall top track, with four 3/4-in. high strength bolts. All attachments of the wall to the test frame were accomplished using a pneumatic wrench.

After a wall was installed in the test frame, displacement transducers were attached to monitor and record the wall performance. The transducers measured and recorded overturning uplift at the bottom of the wall (at each holdown), slip at the bottom of the wall, lateral displacement at the top of the wall and reaction beam displacement (see Figure 2). The resisting load was measured directly by a load cell in line with the load distribution member and the hydraulic ram.

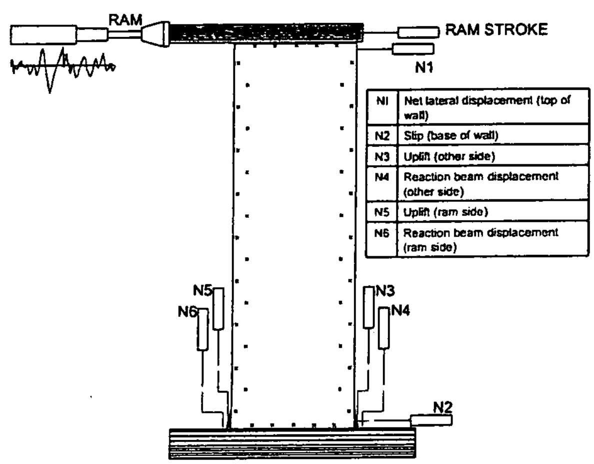


Figure 2. Instrumentation and test setup

The reversed cyclic test procedure used in this program required cycling a wall through a series of specified increasing top wall displacements/drifts (target displacements) up to 2.8 in.. Target displacements and the corresponding number of cycles at each displacement are given in Table 2. Under the current model codes (IBC, UBC and NFPA), the maximum/allowable inelastic drift for an 8 ft. wall height is limited to 2.4 in.. Thus, per Table 2, the incremental displacement from one target displacement to the next was approximately 8 percent of the model codes inelastic drift limit. During a test, the cycling frequency was held constant at 0.2 Hz (or 5 seconds per cycle), and data was sampled and recorded at a rate of 50 samples per seconds (i.e. one sample every 0.02 seconds).

Table 2. Reversed cyclic test procedure

Target Displacement, in.	No. of Cycles	Target Displacement, in.	No. of Cycles
0.2	3	1.8	3
0.4	3	2.0	3
0.6	3	2.2	3
0.8	3	2.4	3
1.0	3	2.6	3
1.2	3	2.8	3
1.4	3		
1.6	3		

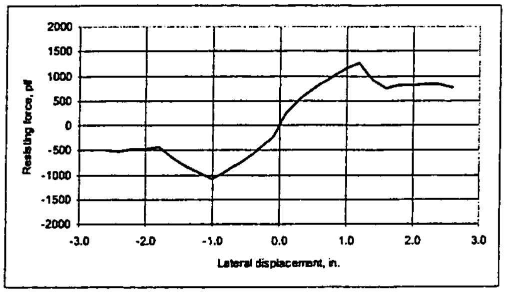
TEST RESULTS

Table 3 summarizes the failure modes, maximum resistances and corresponding lateral displacements/drifts (resistance and displacement are given as the average of the positive (pull) and negative (push) values from the peak response envelope or backbone curves) for the eight wall tests. Figures 3 and 4 show the envelope (backbone) curves derived from the hysteretic response of the sheet steel and OSB walls, respectively. The complete hysteresis response curves are given in Appendix A.

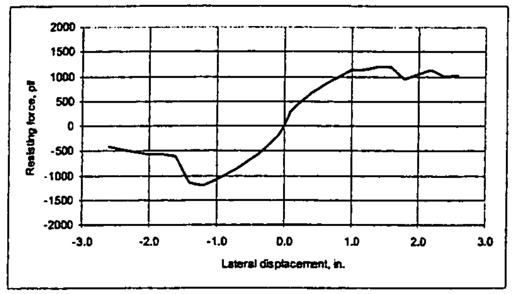
Table 3. Test results

Test No.	General Wall Description [1]	Measured Resistance		Mode of Failure
		Maximum Load [2,3], plf	Total Drift @ Maximum load, in.	
2by8-TA 2by8-TB	2 ft. x 8 ft. wall with 27-mil sheet steel; pins at 3" and adhesive	1165 1207	1.094 1.296	Buckling in the chord (boundary) studs at the web punchout.
4by8-TA 4by8-TB	4 ft. x 8 ft. wall with 27-mil sheet steel; pins at 3"/12" and adhesive	1376 1121	1.092 1.099	Loss of bond between the sheet steel and the adhesive; fastener pullout from the framing.
OSB6-TA OSB6-TB	4 ft. x 8 ft. wall with 7/16-in. OSB; pins at 6"/12" and adhesive	1419 1656	0.699 0.899	In-plane (rolling) shear failure in the OSB; A combination of fastener pullout from the framing, fastener fracture and panel pullover.
OSB12-TA OSB12-TB	4 ft. x 8 ft. wall with 7/16-in. OSB; pins at 12"/12" and adhesive	1200 1532	0.699 0.895	In-plane (rolling) shear failure in the OSB; A combination of fastener pullout from the framing, fastener fracture and panel pullover.

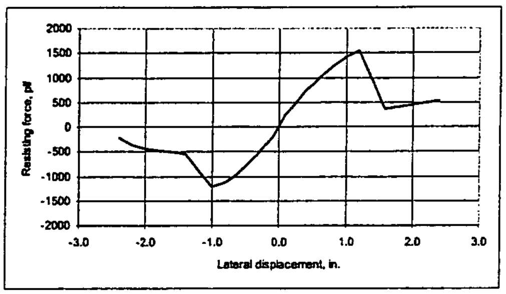
[1] Adhesive applied per Figure 1
[2] Measured resistance in lb. divided by the wall dimension parallel to the applied load
[3] Average of "push" and "pull" resistances



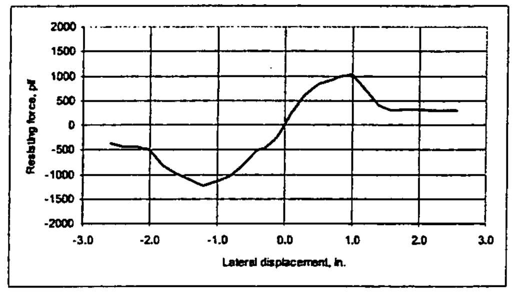
2by8-TA



2by8-TB

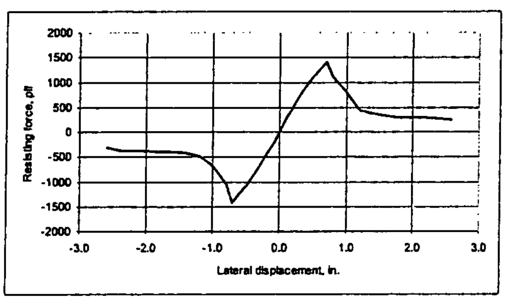


4by8-TA

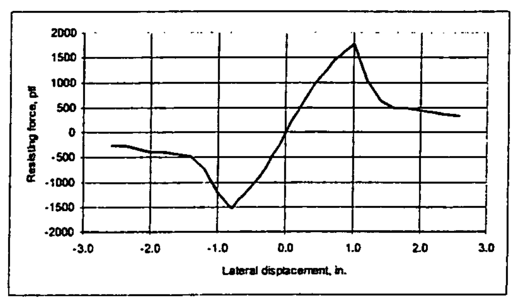


4by8-TB

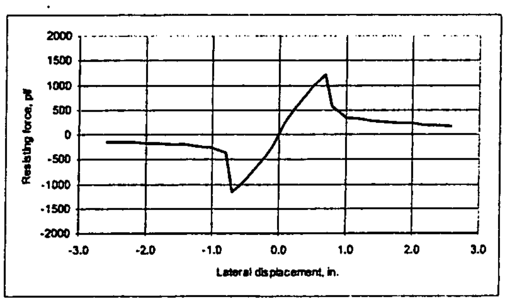
Figure 3. Resistance versus lateral displacement envelope curves for the sheet steel walls



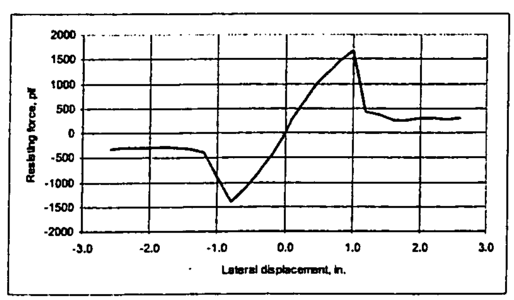
OSB6-TA



OSB6-TB



OSB12-TA

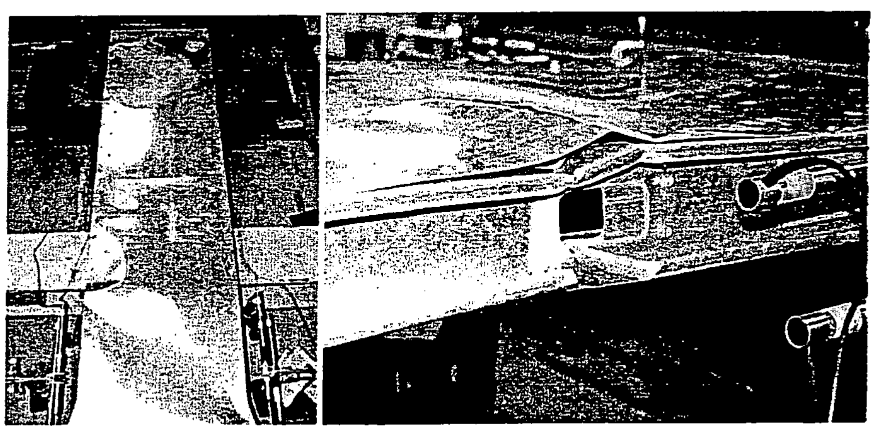


OSB12-TB

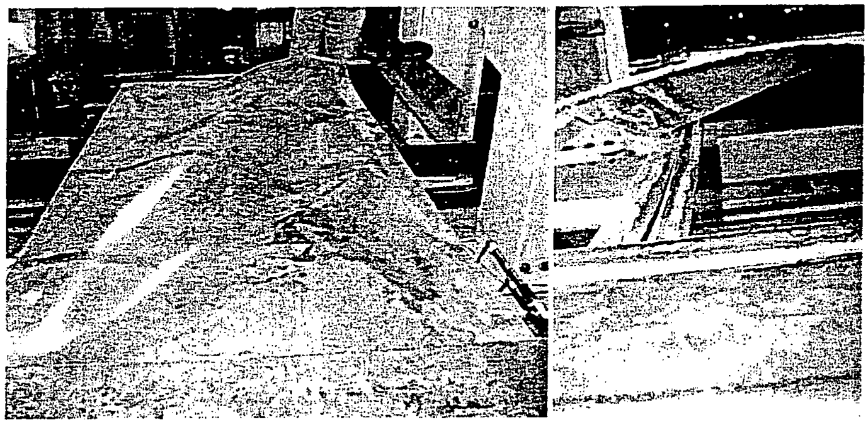
Figure 4. Resistance versus lateral displacement envelope curves for OSB sheathed walls

Sheet Steel Shear Walls: The overall response of the sheet steel walls was characterized by shear buckling and tension field action. In the 4 ft. x 8 ft. walls failure resulted from a loss the bond strength between the structural adhesive and sheet steel as the sheet buckled out of the plane of the wall. This behavior was followed by a progressive pull-out of pins from the framing, including pins at the interior studs. In the 2 ft. x 8 ft. specimens, failure resulted from local buckling in the chord studs at the web punchouts immediately above the holdowns. In this report, failure is defined by a decrease in wall resistance under increased lateral displacement/drift. Figure 5 shows the failure modes for all the sheet steel walls. In one test, 4by8-TA, bending of the top track was observed at one end of the wall. It appears that this behavior resulted from the combined effects of overall twisting of the chord studs, tension field action in the sheet steel and inadequate restraint provided by the round washer used to secure the top track of the wall to the load distribution member, at this end of the wall. When a square washer extending over a larger area of the web track was used, test 4by8-TB, top track bending was eliminated.

OSB Shear Walls: In the OSB walls, failure was observed to result from in-plane (rolling) shear in the structural panel. As shown in Figure 6, the adhesive bonded extremely well to both the steel framing and the OSB. Once bond was lost, a more sudden degradation of wall resistance was observed compared to the sheet steel walls and there was a progressive loss of resistance as a result of pin pullout from the framing, pin fracture and panel pullover.



(a) 2by8 sheet steel walls



(b) 4by8 sheet steel walls

Figure 5. Failure of sheet steel shear walls

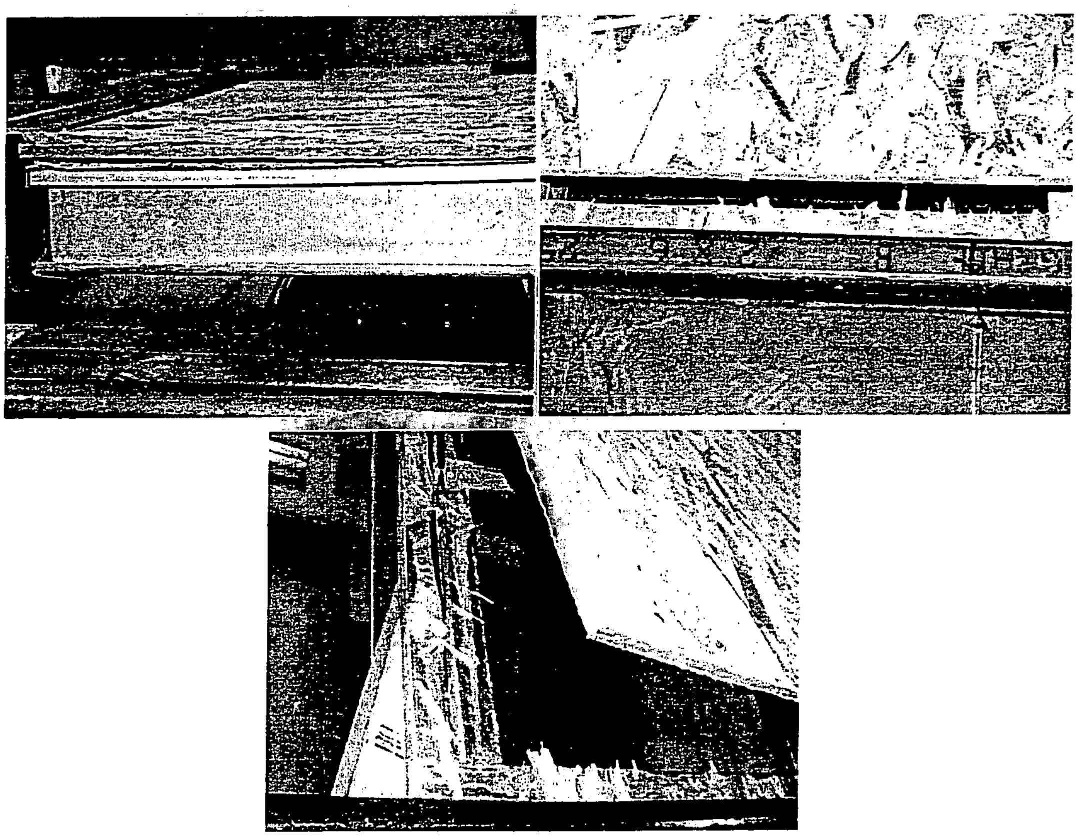


Figure 6. Failure of OSB shear walls

INTERPRETATION and DISCUSSION OF TEST RESULTS

From a design/comparison perspective, one method of interpreting these test results is to use the criteria employed in the development of the seismic design values in the current model codes. In using this approach, it is important to keep in mind the limited number of tests conducted.

The seismic design values for CFS shear walls in the model codes are based on an assumed seismic response modification factor (R) for an expected wall behavior. The recommended design values were then interpreted independent of R. Specifically, the design values in the model codes were developed using a degraded (as opposed to peak) strength envelope as follows:

> The nominal capacity, P_{nom}, of a wall was taken as the lower of the maximum wall resistance, P_{max}, and 2.5 times the wall resistance defined by 0.5 in. of lateral displacement. The LRFD and ASD level capacities were then computed as 0.55 times the nominal capacity and the nominal capacity divided by 2.5, respectively.

Using the above method with the peak (non-degraded) strength envelope (Figures 3 and 4), the nominal, LRFD and ASD level capacities of the tested walls are summarized in Table 4.

Table 4. Interpreted design values

Specimen	P_{nom}, plf	Δ @ P_{nom}, in.	P_{LRFD}, plf	Δ @ P_{LRFD}, in.	P_{ASD}, plf
2by8-TA	*1165*	*1.094*	*641*	*0.433*	
2by8-TB	*1207*	*1.296*	*664*	*0.450*	
2by8 (average)	**1186**	**1.195**	**652**	**0.442**	**474**
4by8-TA	*1376*	*1.092*	*757*	*0.444*	
4by8-TB	*1121*	*1.099*	*616*	*0.396*	
4by8 (average)	**1248**	**1.110**	**686**	**0.420**	**499**
OSB6-TA	*1419*	*0.699*	*781*	*0.338*	
OSB6-TB	*1656*	*0.899*	*911*	*0.402*	
OSB6 (average)	**1537**	**0.799**	**846**	**0.370**	**615**
OSB12-TA	*1200*	*0.699*	*660*	*0.320*	
OSB12-TB	*1532*	*0.895*	*843*	*0.398*	
OSB12 (average)	**1366**	**0.797**	**751**	**0.359**	**546**

Per the data in Table 4, there appears to be no significant difference in capacity of the 2 ft. x 8 ft. and 4 ft. x 8 ft. sheet steel shear walls. Further, given the mode of failure in the 2 ft. x 8 ft. walls, it may be concluded that the capacity of these walls may have been higher if chord stud buckling was prevented (as required by current model codes). When the results for the OSB walls are analyzed, an apparent increase of approximately 12 percent in capacity of the wall is evident for pins are installed at 6 in. on center compared to a wall with pins at 12 in. on center.

A comparison the response curves for the 2 ft. x 8 ft. sheet steel walls in this test program, Figure 3, with the measured peak response of walls where the sheet steel is attached with No. 8 screws only (no structural

adhesive), Figure 7, indicates that use of the adhesive results in a more rapid degradation in resistance after the maximum/peak resistance is attained.

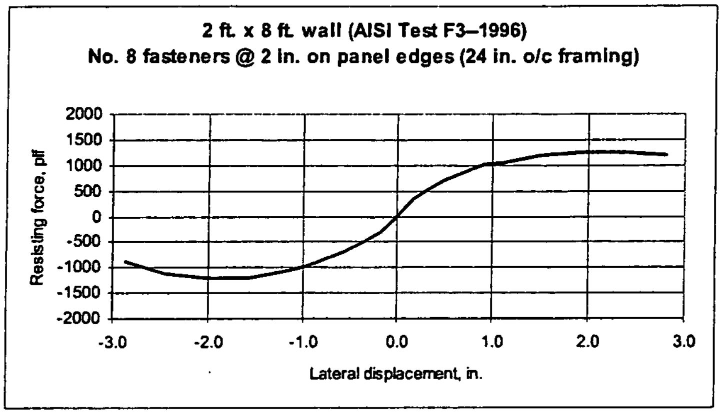


Figure 7. 27-mil 2 ft. x 8 ft. shear wall test from earlier AISI research (Serrette et al. 1997)

A comparison of the 2by8 wall performances (before buckling in the chords) with those of the 4by8 walls (see Figure 8) suggests that the stiffness of the narrower 2by8 walls was roughly the same as the 4by8 walls. One important observation made in the 2by8 tests was the fact fracture of the buckled studs from repeated reversal of load with increasing lateral displacements occurred (6 to 8) cycles after initial stud buckling.

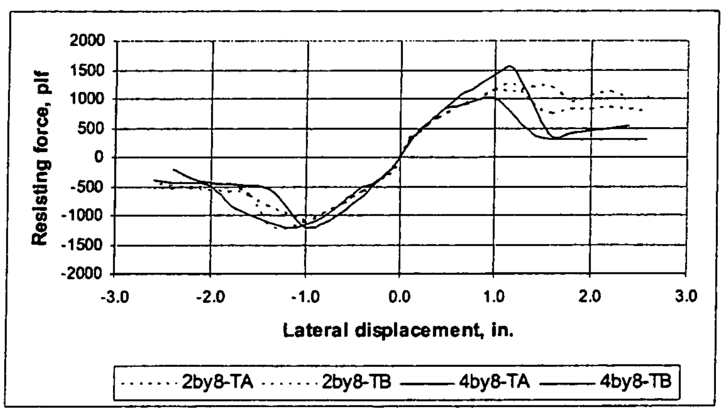


Figure 8. Comparison of sheet steel test results

An inspection of the response curves for the OSB walls indicates that the overall behavior of these walls was essentially linear elastic up to the nominal strength of the wall and there was no difference in wall stiffness for the two pin schedules. Further, although there was a rapid degradation of post-peak resistance, these walls were capable of maintaining a reduced or residual strength in the range of the ASD capacities in Table 4 (at lateral displacements exceeding 1.50 times the displacements at nominal strength). When

evaluating the significance of these residual strength values it is important to note that at both ends of the wall there was a small gap between the structural panel and the test frame that permitted bearing of the sheathing on the reaction frame after the peak resistance was attained.

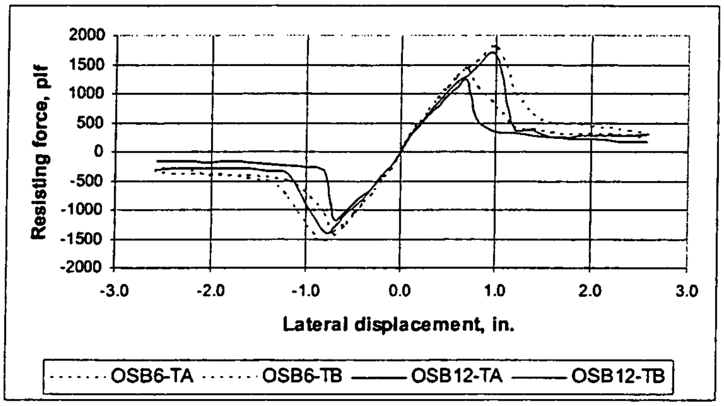


Figure 9. Comparison of OSB test results

Finally, Table 5 compares the recommended design values for these tests (Table 4) with values for similar (not identical) systems, as published in the 2003 IBC. The test to IBC values ranged from 1.04 to 2.20 suggesting that the structural adhesive application with steel pins may be a viable method for developing lateral resistance in cold-formed steel frame shear walls. For seismic design, further refinements to the interpretation of test data may be required given the rapid degradation in post-peak strength seen in these tests. These refinements will be more significant for areas of high seismicity (SDC D, E and F).

Table 5. Comparison of test data with 2003 IBC design values

Test No.	Wall Description	Nominal Resistance, plf		Test/2003 IBC
		2003 IBC [1]	Test	
2by8	Sheet steel sheathed wall with screws fasteners at 3 in. on panel edges	543 [2,4,5] (597) [3,4,5]	1186	2.18 (1.99)
4by8	Sheet steel sheathed wall with screws fasteners at 3 in. on panel edges and 12 in. in the field	1085 [2,4] (1194) [3,4]	1248	1.15 (1.04)
OSB6	OSB sheathed wall with screws fasteners at 6 in. on panel edges and 12 in. in the field	700 [2] (770) [3]	1537	2.20 (2.00)
OSB12	Not permitted in the 2003 IBC	--	1366	---

[1] IBC values are for applications with No. 8 self-drilling screw fasteners
[2] IBC values are based on a degraded strength
[3] IBC values increased 10% (conservatively) for expected peak (non-degraded) resistance
[4] Values interpreted, by linear interpolation, from 2 in./12 in. and 4 in./12 in. fastener schedules
[5] 50% reduction of 2:1 aspect ratio wall value for 4:1 aspect ratio wall

CONCLUSION

A series of eight shear walls (four sheet steel walls: two 2 ft. x 8 ft. walls and two 4 ft. x 8 ft. walls; and four 4 ft. x 8 ft. OSB walls) were tested to evaluate the reversed cyclic performance of cold-formed steel shear walls with structural sheathing attached using a combination of steel pin fasteners and a structural adhesive. Overall, except for the 2 ft. x 8 ft. sheet steel shear walls, the maximum resistances were governed by failure due to a degradation of the bond at the framing-sheathing interface. The 2 ft. x 8 ft. walls failed by buckling in the chord studs at the web punchouts above the holdowns.

The measured resistances exceeded values in the current model codes for similarly sheathed walls (sheathing attached with screw fasteners only). For the OSB walls, the measured responses up to the maximum wall resistances were approximately linear and this behavior was followed by a sudden degradation in strength. The sheet steel walls exhibited a more nonlinear behavior with a less severe reduction in strength after the maximum resistance. Based on these test results, the use of structural adhesives with pneumatically driven steel pins appears promising.

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IBC, 2003. 2003 International Building Code, International Code Council, Inc., Country Club Hills, IL.

NFPA, 2003. NFPA 5000 Building Construction and Safety Code, National Fire Protection Association, Quincy, MA,

Serrette, R. et. al., 1997. "Additional Shear Wall Values for Light Weight Steel Framing," Light Gauge Steel Research Group, Research Report No. LGSRG-1-97, Santa Clara, CA, March.

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APPENDIX A

Hysteresis Curves

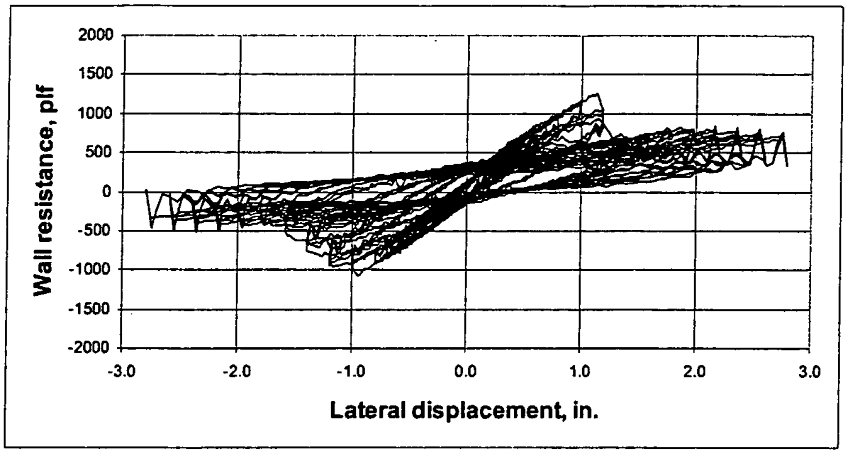

Figure A1. Hysteresis response curve for Test 2by8-TA

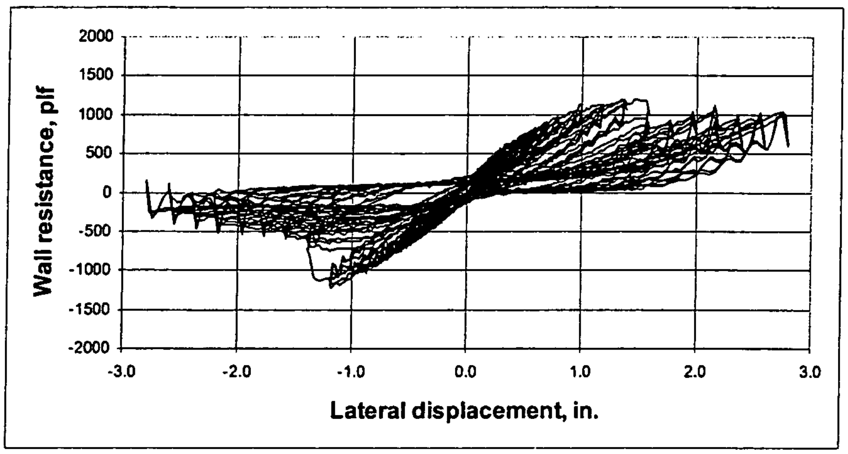

Figure A2. Hysteresis response curve for Test 2by8-TB

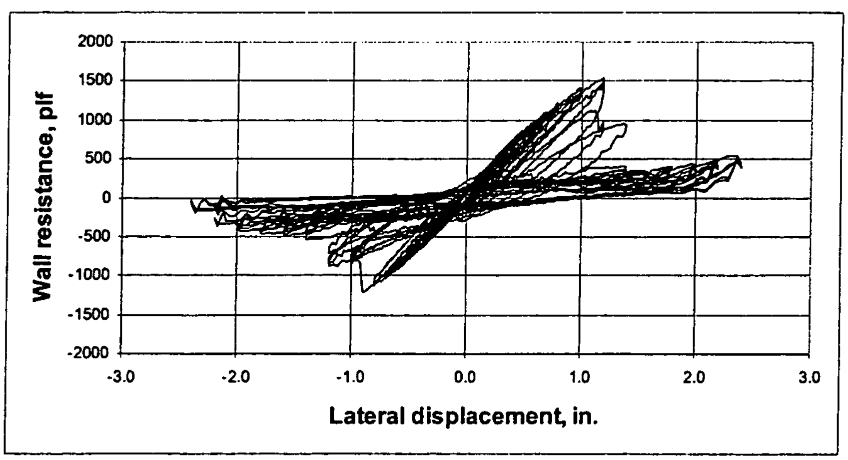

Figure A3. Hysteresis response curve for Test 4by8-TA

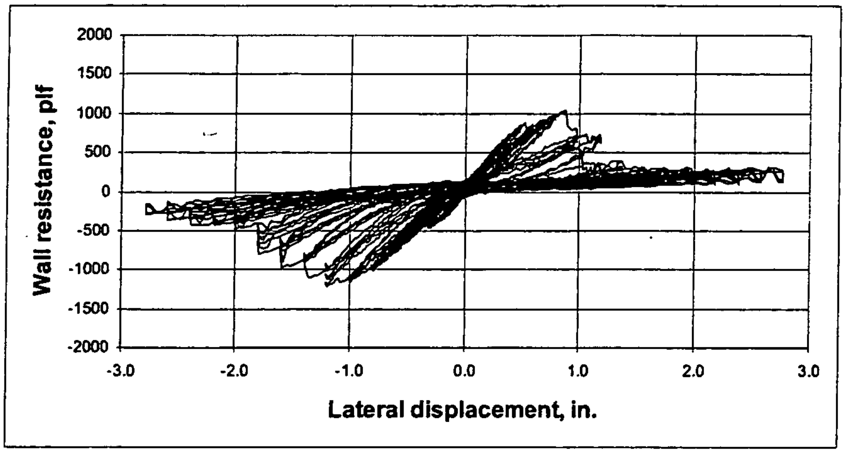

Figure A4. Hysteresis response curve for Test 4by8-TB

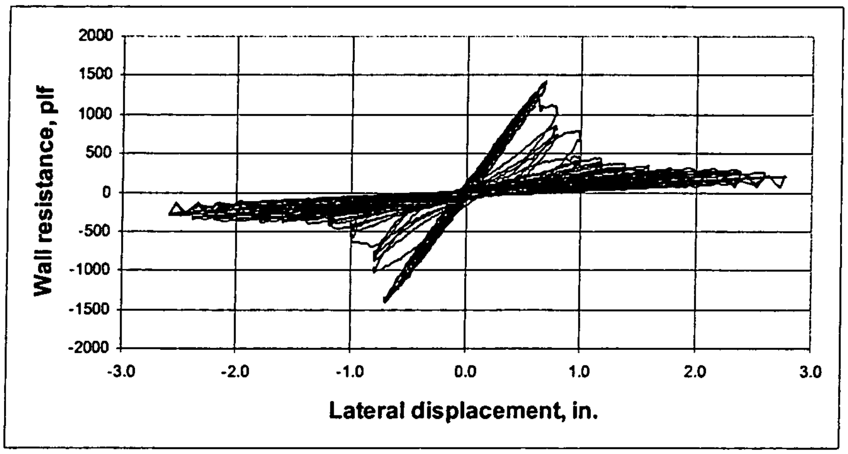

Figure A5. Hysteresis response curve for Test OSB6-TA

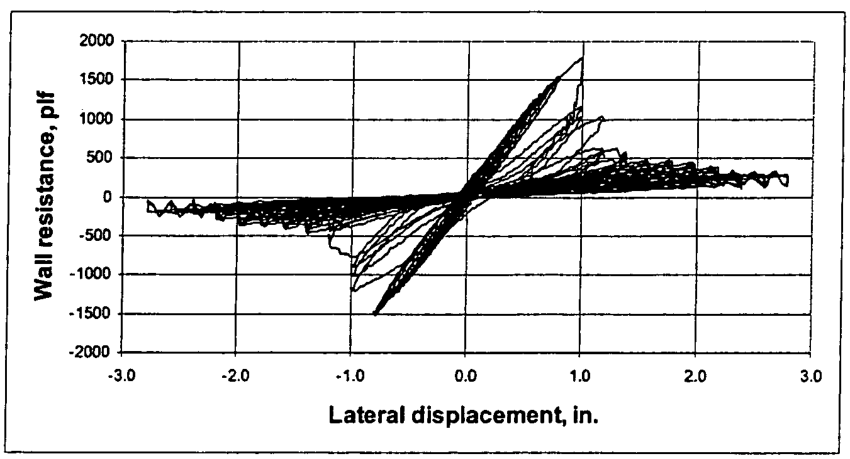

Figure A6. Hysteresis response curve for Test OSB6-TB

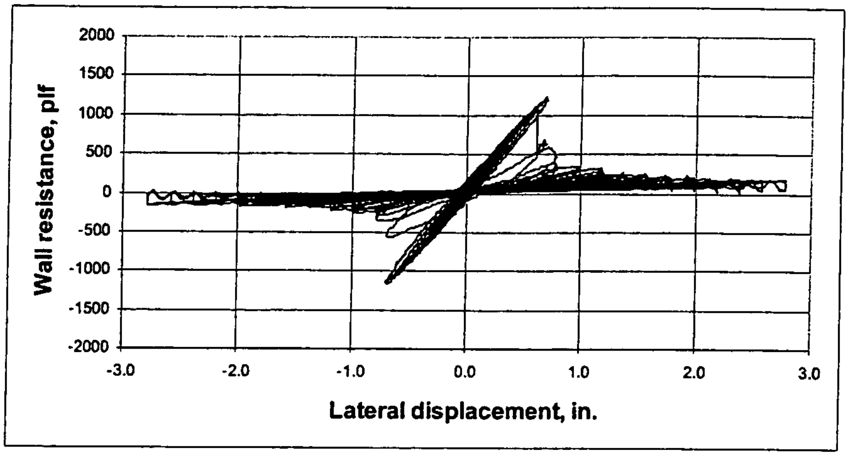

Figure A7. Hysteresis response curve for Test OSB12-TA

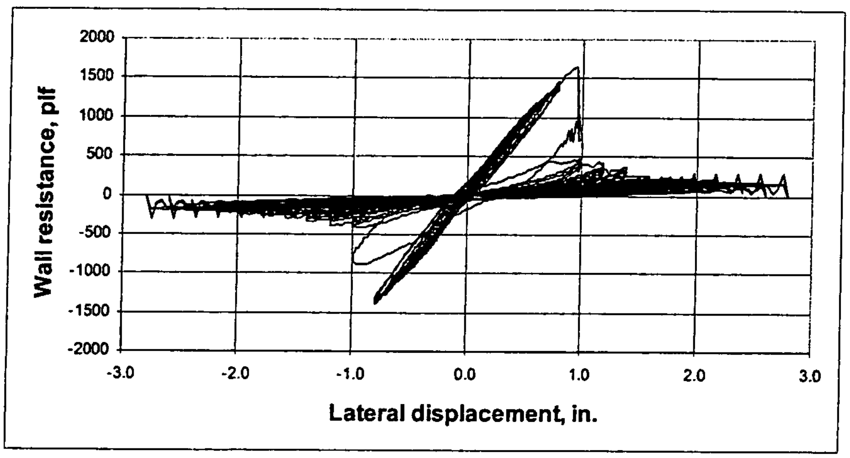

Figure A8. Hysteresis response curve for Test OSB12-TB